Wind and Seismic Effects

Proceedings of the 30th Joint Meeting

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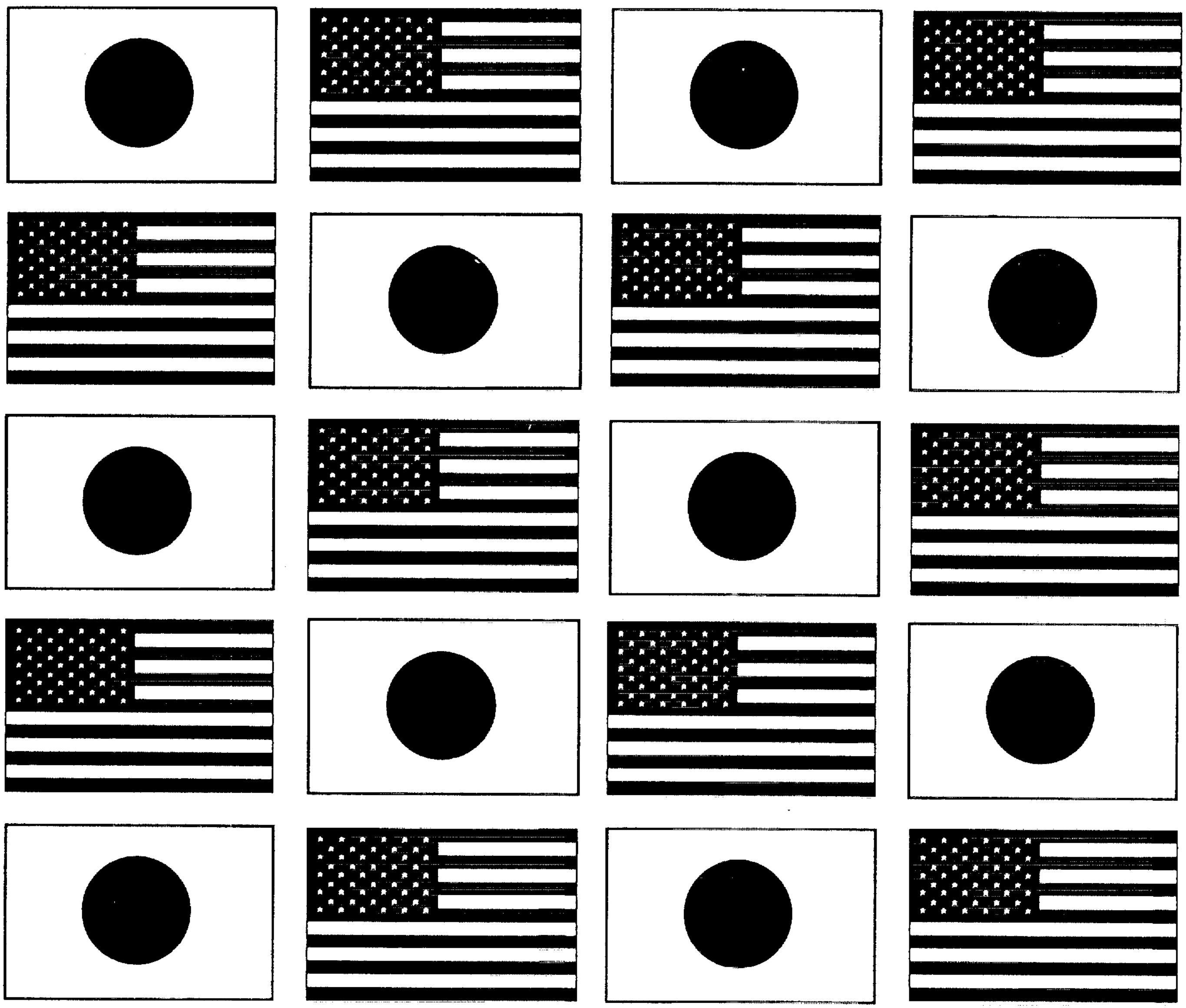

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Wind and Seismic Effects

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Design Guidelines for the Seismic Modification of Welded Steel Moment Frame Buildings

by

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ABSTRACT

Prompted by the widespread damage to welded beam-to-column connections as a result of the 1994 Northridge earthquake, the National Institute of Standards and Technology (NIST) initiated a study to determine the efficacy of various techniques for the modification of existing welded steel moment frame connections to improve their seismic performance. Problems associated with retrofitting of existing WSMF buildings are discussed and the alternative strategies selected for study are described. Results of 18 full-scale tests of two-sided (interior) connections, some having a lightweight concrete slab on metal deck, are presented. Design guidelines, based on the experimental results, have been written in cooperation with the American Institute of Steel Construction (AISC) and are described.

KEYWORDS: buildings; connections; design; earthquake engineering; frames; rehabilitation; steel

1. INTRODUCTION

The January 17, 1994 Northridge earthquake caused significant unexpected damage to welded steel moment frame (WSMF) construction. In particular brittle fractures in the beam-to-column moment connections were found in over 100 buildings (Youssef et al. 1995). In some instances the beam flange-to-column flange welds fractured completely resulting in greatly reduced moment resistance at the connection. Fortunately, no members or buildings collapsed as a result of the connection failures and no lives were lost. Nevertheless, the widespread occurrence of these connection failures is indicative of basic deficiencies in welded steel moment frame design and construction practice prior to the 1994 Northridge earthquake. Indeed, the beam-to-column connection detail in common use at the time of the Northridge earthquake was deleted in an emergency code change ("ICBO Board" 1994). There is concern that existing structures incorporating these pre-Northridge welded moment connections may not provide the desired level of seismic performance in a major earthquake. The work described in this paper addresses the design of modifications to the WSMF connections to provide improved seismic performance.

1.1 Background

In the United States, seismic design of WSMF construction is based on the assumption that, in a severe earthquake, frame members will yield in a ductile fashion thereby dissipating energy. The welds and bolts that connect the frame members, being considerably less ductile, must be designed so as not to fracture (SEAOC 1990). The beam-to-column moment connections should be designed, therefore, for either the strength of the beam in flexure or the moment corresponding to the joint panel zone shear strength.

The Uniform Building Code, or UBC (ICBO 1994), is adopted by nearly all California jurisdictions as the standard for seismic design. From 1988 to 1994 the UBC prescribed a beam-to-column connection that was deemed to satisfy the above strength requirements. This "prescribed" detail required that the beam flanges be welded to the column using complete joint penetration (CJP) groove welds.

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The beam web connection was made by either welding directly to the column or by bolting to a shear tab which in turn was welded to the column. A version of this prescribed detail is shown in Figure 1. Although this connection detail was first prescribed by the UBC in 1988, it has been widely used since the early 1970's.

In general, fractures of these "prescribed" moment connections were found to initiate at the root of the beam flange CJP weld and propagate through either the beam flange, the column flange, or the weld itself. In some instances, fracture extended through the column flange and into the column web. The steel backing, which was generally left in place, produced a mechanical notch at the weld root. Fractures often initiated from weld defects in the root pass which were contiguous with the notch introduced by the weld backing. A schematic of a typical fracture path is shown in Figure 2. A fracture analysis, based upon measured material properties and measured weld defect sizes (Kaufmann et al. 1997), revealed that cleavage fracture would occur at stress levels roughly equal to the nominal yield strength of the beam.

1.2 Connection Failures

Brittle fracture will occur when the applied stress intensity, which can be computed from the applied stress and the size and character of the initiating defect, exceeds the critical stress intensity for the material. The critical stress intensity is in turn a function of the fracture toughness of the material. In the fractures that occurred in WSMF construction as a result of the Northridge earthquake, several contributing factors were observed which relate to the fracture toughness of the materials, size and location of defects, and magnitude of applied stress. These factors are discussed here.

The self-shielded flux cored arc welding (FCAW-SS) process is widely used for the CJP flange welds in WSMF construction. The E70T-4 electrode in common use prior to the Northridge earthquake, is not rated for notch toughness and has been found to have very low Charpy V-notch (CVN) toughness, frequently on the order of 7 J at 20° C (Kaufmann et al. 1997).

The practice of leaving the steel backing in place introduces a mechanical notch at the root of the flange weld joint as shown in Figure 2. Also, weld defects in the root pass, being difficult to detect using ultrasonic inspection, may not have been rejected by the inspectors and therefore were not repaired. Further, the use of "end dams" in lieu of weld tabs was widespread.

The weld joining the beam flange to the face of the relatively thick column flanges is highly restrained. This restraint inhibits yielding and results in somewhat more brittle behavior. Further, the stress across a beam flange connected to a wide flange column section is not uniform but rather is higher at the center of the flange and lower at the flange tips. Also, when the beam web connection is bolted rather than welded, the moment is carried principally by the beam flanges. Finally, the actual yield strength of a steel member may exceed the nominal yield strength by a considerable amount. Since seismic design of moment frames relies on beam members reaching their plastic moment capacity, an increase in the yield strength translates to increased demands on the CJP flange weld.

Modifications to pre-Northridge WSMF connections to achieve improved seismic performance seek to reduce or eliminate some of the factors which contribute to brittle fracture mentioned above. Methods of achieving improved seismic performance are addressed next.

2. IMPROVING SEISMIC PERFORMANCE

There are several approaches to minimizing the potential for fracture including, 1) strengthening the connection thereby reducing the beam flange stress at the column face, 2) limiting the beam moment at the column face, or 3) increasing the fracture resistance of welds. Any of these basic approaches, or a combination of them, may be used. This paper presents three connection modification methods: welded haunch, bolted bracket, and reduced beam section. The first two of these modification methods employ the approach of strengthening the connection and thereby forcing inelastic action to take place in the beam section away from the face of the column and the CJP flange welds. The third method seeks to limit the moment at the column face by reducing the beam section at some distance from the column thereby introducing a structural "fuse."

2.1 Reduced Beam Section

The reduced beam section (or RBS) technique is illustrated in Figure 3. As shown, the beam flange is reduced in cross section thereby weakening the beam in flexure. Various profiles have been tried for the reduced beam section but only the circular cut is considered here. The intent is to force a plastic hinge to form in the reduced section. By introducing a structural "fuse" in the reduced section, the force demand that can be transmitted to the CJP flange welds is also reduced. The reduction in beam strength is acceptable in most cases since drift requirements frequently govern moment frame design and the members are larger than needed to satisfy strength requirements. The RBS technique applied to the bottom flange only tends to reduce the overall frame stiffness on the order of 5%. This technique has been shown to be promising in tests intended for new construction.

The RBS plays a role quite similar to that of connection reinforcement schemes such as cover plates. Both the RBS and connection reinforcement move the plastic hinge away from the face of the column and reduce stress levels in the vicinity of the CJP flange welds. Connection reinforcement often requires welds that are difficult and costly to make and inspect. These problems are lessened with the RBS, which is somewhat easier to construct. On the other hand, a greater degree of stress reduction can be achieved with connection reinforcement. With the RBS, there is a practical limit to the amount of flange material that can be removed and consequently, there is a limit to the degree of stress reduction that can be achieved with the RBS.

The reduced beam section appears attractive for the modification of existing connections because of its relative simplicity, and because it does not increase demands on the column and panel zone. For new construction, RBS cuts are typically provided in both the top and bottom beam flanges. However, when modifying existing connections, making an RBS cut in the top flange may prove difficult due to the presence of a concrete floor slab. Consequently, in this study addressing the modification of existing connections, the RBS cut is provided in the bottom flange only.

2.2 Welded Haunch

Welding a tapered haunch to the beam bottom flange (see Figure 4) has been shown to be very effective for enhancing the cyclic performance of damaged moment connections (SAC 1996) or connections for new construction (Noel and Uang 1996). The cyclic performance can be further improved when haunches are welded to both top and bottom flanges of the beam (SAC 1996) although such a scheme requires the removal of the concrete floor slab in existing buildings. Reinforcing the beam with a welded haunch can be viewed as a means of increasing the section modulus of the beam at the face of column. However, a more appropriate approach is to treat the flange of the welded haunch as a diagonal strut. This strut action drastically changes the force transfer mechanism of this type of connection and can be shown to greatly reduce the stress in the beam flange welds.

The tapered haunch is usually cut from a structural tee or wide flange section although it could be fabricated from plate. The haunch web is fillet welded to the beam and column flanges. The haunch flange is then groove welded to the beam and column flanges (see Figure 4).

2.3 Bolted Bracket

The bolted bracket is an alternative to the welded haunch and has the added advantage that no field welding is required. Rather, high strength bolts are used to attach a welded steel bracket, fabricated from plate, to both the beam and column as shown in Figure 5. Installation of the bolted bracket eliminates the problem associated with welding such as venting of welding fumes, supply of fresh air, and the need for fire protection.

As with the welded haunch, the bolted bracket forces inelastic action in the beam outside the reinforced region. Tests have shown this to be an effective repair and modification technique producing a rigid connection with stable hysteresis loops and high ductility (Kasai et al. 1997).

Various types of bolted bracket have been developed. The haunch bracket (Figure 5) consists of a shop-welded horizontal leg, vertical leg and vertical stiffener. The two legs are bolted to the beam and column flanges. An angle bracket (not shown), cut from a relatively heavy wide flange section, has been used for the top flange where it is desirable to conceal the modification within the concrete slab.

3. EXPERIMENTAL PROGRAM

The modification of pre-Northridge moment connections differs from new construction in two significant ways:

1) Existing welds are generally of low toughness E70T-4 weld metal with steel backing left in place and their removal and replacement using improved welding practices and tougher filler metal is both difficult and expensive, and

2) Access to the connection may be limited, especially by the presence of a concrete floor slab which may limit or preclude any modifications to the top flange.

With these limitations in mind, the National Institute of Standards and Technology (NIST) initiated an experimental program for the express purpose of determining the connection performance for various levels of connection modification. As such, initial tests were conducted on specimens that typically involved modifications only to the bottom flange. Based on successes and failures, additional remedial measures were applied until acceptable performance levels were obtained.

The NIST experimental program was designed to complement other test programs that had been completed or were in progress. In the majority of the tests conducted prior to NIST involvement, the test specimens consisted of bare steel frame subassemblages representing one-sided (exterior) connections. The NIST program sought to obtain data on interior, or two-sided, connections to determine if such connections perform as well as one-sided connections. Additionally, the presence of a concrete slab, whether designed to act compositely or not, tends to shift the elastic neutral axis of the beam upward thereby increasing tensile flexural strains at the bottom beam flange weld as compared to those in a bare steel frame. To address this issue, some NIST tests included a steel deck-supported lightweight concrete slab. The concrete slab was not designed for composite action, however, shear studs designed to transfer lateral forces into the moment frame force the slab to act compositely with the steel beam.

Beam sections used in the NIST experimental program were selected to conform to those used in the SAC Phase 1 test program (SAC 1996). Two-sided connections, however, required larger columns than those used in the SAC tests to accommodate the unbalanced beam moments. Columns were selected so as to not require the addition of column web stiffening, commonly referred to as "doubler plates." The columns selected also did not require continuity plates as would be consistent with practice in the early 1980's. The two test specimen sizes consisted of the following beam and column sections, respectively: W30x99, W12x279 and W36x150, W14x426.

The NIST experimental program involved the testing of 18 full-size beam-to-column connections which had been modified using the techniques described herein. One specimen was repaired and re-tested. A diagram of the test specimens and representative test apparatus is shown in Figure 6. The tests were conducted at the University of Texas at Austin, the University of California, San Diego and Lehigh University's ATLSS Research Center.

4. EXPERIMENTAL RESULTS

The objective of modifying moment connections in existing buildings is to improve their performance in future earthquakes. Experimental results indicate that the modified connections are generally capable of developing at least 0.02 radian of plastic rotation. A plot of the moment at the face of the column versus total plastic rotation for a representative haunch modification with concrete slab is shown in Fig. 7. While not meeting new construction standards, these modified connections will provide a significant improvement in performance compared to existing pre-Northridge connections. The use of these modified connections should reduce potential economic losses and mitigate safety concerns for existing WSMFs in future earthquakes. A minimum plastic rotation of 0.02 radian provides a reasonable and realistic basis for the seismic rehabilitation of a wide variety of ordinary buildings constructed with WSMFs.

4.1 RBS Results

The experimental data indicate that the addition of the beam flange cutout, by itself, is not sufficient to significantly improve connection performance. In all cases in which the low toughness beam flange groove welds were not replaced, the welds fractured at low levels of plastic rotation indicating that additional measures would be required to significantly improve performance. Better performance was achieved by not only providing a flange cutout, but also by replacing the existing top and bottom beam flange groove welds with a higher toughness weld metal. Tests indicated that this level of modification permits the development of plastic rotations on the order of 0.02 radian to 0.025 radian.

4.2 Welded Haunch Results

For both sets of member sizes tested, the test data show that, when the beam top flange groove welded joint was left in its pre-Northridge condition, the welded haunch modification outperformed the RBS modification. Of the three sets of bare steel specimens tested, five beams experienced welded fracture at the top flange. Two-thirds of the beams, however, were able to develop plastic rotations of at least 0.025 radian. When the concrete slab was present, none of the beams experienced weld fracture and the plastic rotation achieved for the six beams tested varied from 0.028 radian to 0.031 radian, more than adequate for modification purposes.

4.3 Bolted Bracket Results

For the bolted bracket tests, four specimens showed early fracture of the top flange weld when the top flange connection was not modified. When a stiff double angle was bolted to the top flange and column face, the top flange did not fracture and the connections were able to develop 0.05 radian plastic rotation. The addition of a top angle, however, necessitates the removal of a portion of the concrete slab.

5. DESIGN GUIDELINES

Based on the results of 18 tests conducted under the program described herein, design guidelines have been developed which allow the proportioning and detailing of the three modification techniques described above. The guidelines have been written in cooperation with the American Institute of Steel Construction (AISC) and will be available as part on their Design Guide series.

The design procedures account for the expected material yield strength and the design plastic moment for each modification has been calibrated to experimental results through a factor which accounts for strain hardening. A plot of this strain hardening factor versus story drift ratio for the RBS modification (W36x150 beams) is shown in Fig. 8. Because the seismic modification of existing welded steel moment connections is not fully investigated and there are many options to consider, the guideline contains a significant amount of commentary. Additionally, the guideline reports, in a common format, the results of 47 tests conducted by others.

6. CONCLUSIONS

This paper has provided a brief overview of the issues surrounding the modification of existing welded steel moment connections to improve their seismic performance. Three modification schemes were described; the welded haunch, bolted bracket and reduced beam section. An experimental program involving 18 tests of full-scale beam-to-column subassemblages was described. Results indicated that the modified connections are capable of developing a minimum of 0.02 radian of plastic rotation. Recommendations for the proportioning and detailing of the various connection modifications are contained in a design guide.

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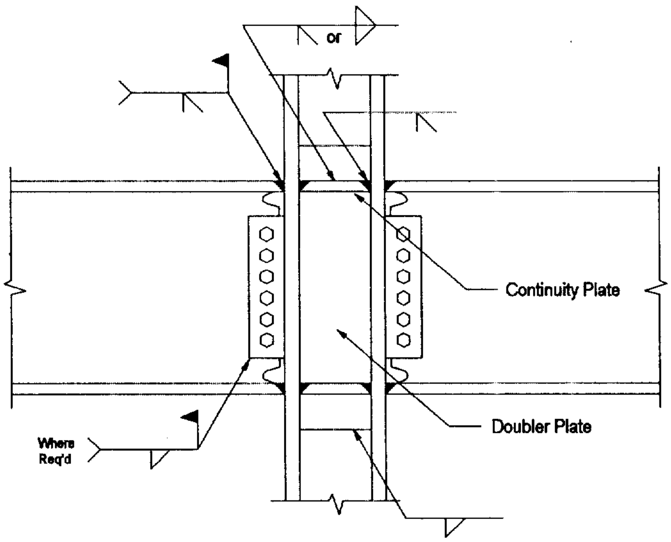

Figure 1 - "Prescribed" Moment Connection Detail

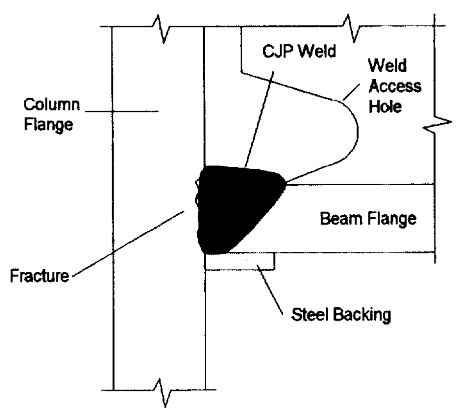

Figure 2 - Fracture of Complete Joint Penetration Flange Weld

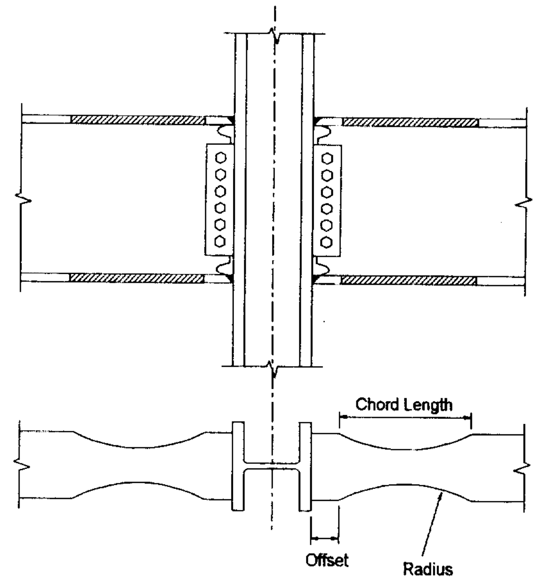

Figure 3 - Reduce Beam Section (RBS) Modification

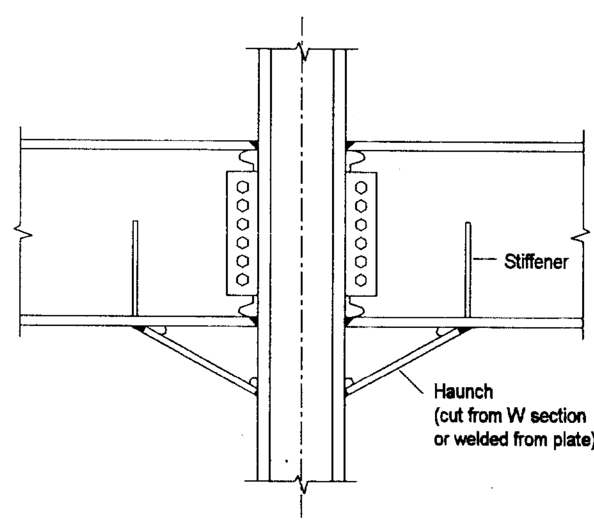

Figure 4 - Welded Haunch Modification

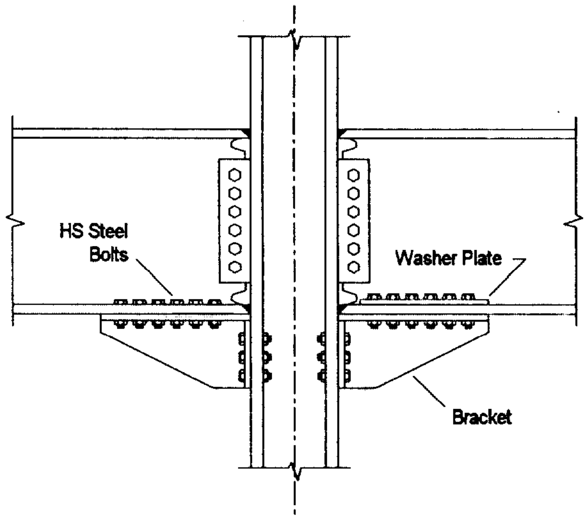

Figure 5 - Bolted Bracket Modification

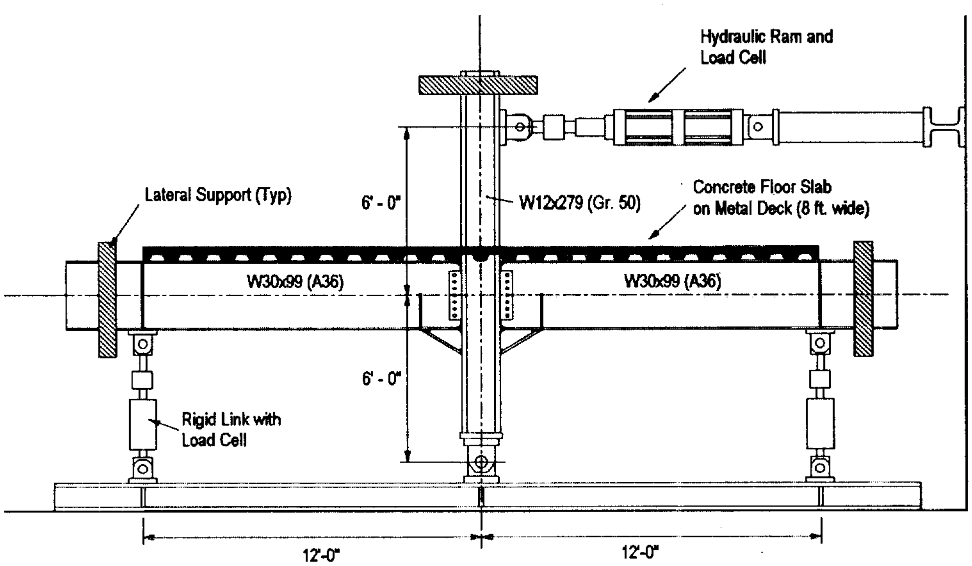

Figure 6 - NIST/AISC Test Set-Up

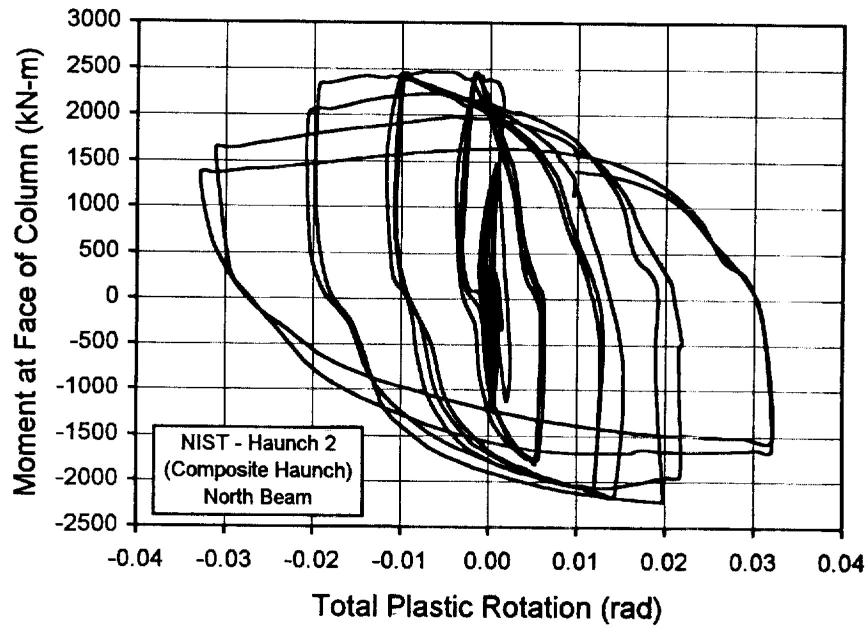

Figure 7 - Moment vs. Plastic Rotation for Haunch Modification

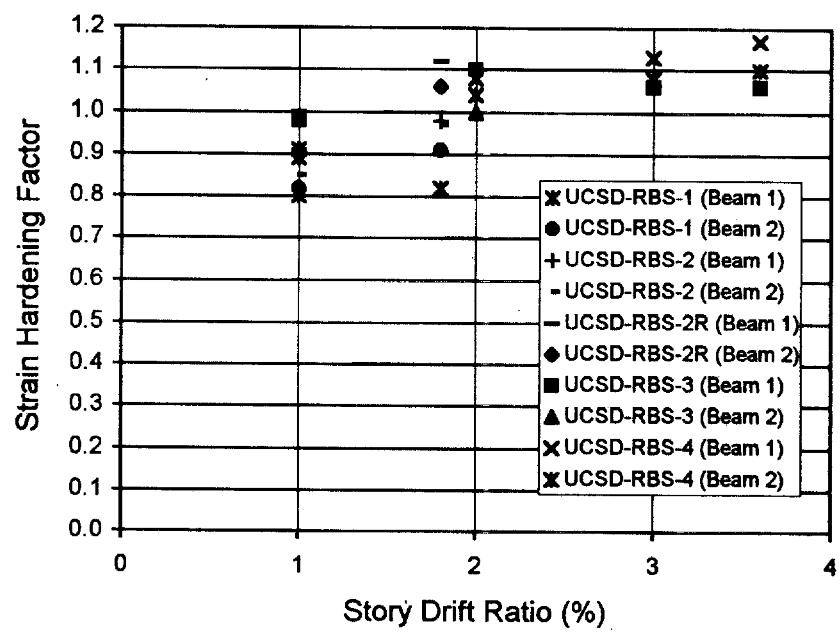

Figure 8 - Strain Hardening Factor vs. Story Drift Ratio for RBS Modification of W36x150 Beams